APPENDIX F

DESIGN EXAMPLES — STRUCTURES

F-1. Introduction

This appendix gives illustrative examples for evaluating and upgrading various types of *lateral* systems in accordance with the criteria and procedures of this manual.

F-2. Use of appendix

The design examples are purely advisory; they are not intended to place super-restrictions on the manual. This appendix is not a handbook for the inexperienced designer. Neither the manual nor the manual supplemented by the appendices can replace good engineering judgment in specific situations. Designers are urged to study the entire manual. Following is a listing of the design examples.

Fig. No.	*Description of Design Examples*
F-1	Sample screening and evaluation of a large military installation.
F-2	Brick building with concrete framing system. A 3-story concrete frame structure with brick exterior walls.
F-3	Building with steel ductile moment-resisting frames and steel braced frames. A 3-story building with transverse ductile moment-resisting frames and longitudinal frames with chevron bracing.
F-4	Building with concrete moment-resisting frames and shear walls. A 10-story building with reinforced concrete lateral force resisting frames in the longitudinal direction and shear walls in the transverse direction.

DESIGN EXAMPLE F-1

SAMPLE SCREENING AND EVALUATION OF A LARGE MILITARY INSTALLATION:

Purpose. This example is presented to illustrate the screening and preliminary evaluation procedures described in chapters 2, 3, and 4 of this manual. For purposes of this example it is assumed that an A/E firm has been contracted to perform the seismic vulnerability evaluation of a military installation. The A/E's contact at the installation are representatives of the Department of Public Works (DPW).

Description of Facility. Military installation with a large inventory of buildings. The data base inventory list includes over 100 structures ranging from flag-poles and gate houses to large warehouses and a regional medical center. The installation is located in the BDM seismic zone 3 and the SDG ground motion specification is equivalent to an ATC 3-06 spectra with $A_a = A_v = 0.30g$. The soil profile coefficient for the site is type S_2.

Inventory Reduction. A meeting of representatives of the A/E and the DPW is held to review the data base inventory list, to establish an inventory reduction procedure, and to visit the site for a general overall visual inspection of the installation. The data base inventory contains data on replacement costs, year of construction, size of building in square feet and number of stories, building identification by number and name, and general usage category. A computer program is able to reorder the data base files according to (1) largest to smallest replacement costs, (2) oldest to newest year built, and (3) largest to smallest building size in square feet.

a. A list of all buildings less than 500 square feet and all buildings with replacement costs less than $50,000 are reviewed to determine if any buildings on the list are categorized as essential or high-risk. Except for the essential and high-risk buildings, all other buildings on this list are removed from the overall inventory list.

b. A list of all buildings used for housing is reviewed. One- and two-family housing, two stories or less, are removed from the overall inventory list.

c. A list of all one-story buildings is reviewed for wood frame and pre-engineered metal construction. Construction type is not listed on the data base, therefore, a visual inspection of listed buildings is required to identify the wood frame and pre-engineered metal buildings for removal from the overall inventory list.

d. The visual inspection during the site visit is also used to list sheds and other low-risk buildings that are seldom occupied by persons (maximum occupancy less than 5 occupants). Unless these structures have an essential function, they are removed from the overall inventory list.

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Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 1 of 12)

e. A list of all buildings constructed since 1983 is reviewed. Essential buildings remain on the overall list and all others are removed from the list when it appears that 1982 BDM criteria or equivalent have been satisfied. When in doubt, structures were kept on the overall inventory list for review in phase I, preliminary screening.

f. A list of the remaining buildings is printed out with the available descriptive information contained in the data base in the order of their identification number for use in the preliminary screening procedure.

Preliminary Screening. By means of the inventory reduction process the number of structures requiring preliminary screening has been reduced from over 100 to 74. A meeting of the A/E representatives and using agency personnel is held to determine the classification (e.g., essential, high-risk, all others) of all structures on the reduced inventory list. The A/E is given access to available design data. This includes the data retrieval files that list available building drawings, storage files of original building drawings, and additional files for available calculations and specifications. Copies of the map of the installation, with building locations, are made available.

a. A table listing the 74 buildings is made with pertinent available data including (1) building classification (essential, high-risk, or all others), (2) construction category (steel, concrete, masonry, wood, and special structures), (3) size, (4) year constructed, and (5) location on the site.

b. All buildings are located on the installation map. The map is divided into 5 geographical zones that include no more than 20 buildings each. This is done in preparation for the field inspection survey. Preliminary screening forms are filled in with data available prior to the field inspection (sample shown on sheet 4). The field surveys are scheduled to cover one of the 5 geographical zones each day. Preliminary screening forms and other data are prepackaged to aid field inspection record taking.

c. During the field surveys the screening forms and additional notes are made to record pertinent observations or information received by the building supervisor or other building personnel (sample shown on sheet 5). One or two photographs of the exterior of the building are obtained, when possible, for inclusion in the report.

d. After the field surveys are completed, the observations are reviewed and compared to data available prior to the site visit. The files of available data are reevaluated to resolve conflicts or to clarify observations made during the field investigation.

e. On the basis of the collected data, the buildings are divided into two groups: (1) those buildings determined not requiring further analysis and (2) those recommended for preliminary evaluation.

(1) Buildings not required for further evaluation are listed in a summary report that includes reasons for making the decision and gives recommendations, if any, for further action.

Sheet 2 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 2 of 12)

(2) Buildings recommended for preliminary evaluation are listed in a summary report that includes a description of the lateral force resisting system, general condition of the structure, observed hazards (if any), and additional comments.

Preliminary Evaluation. The inventory list for the preliminary evaluation has been reduced to 50 structures. The capacities of these structures are estimated by means of a rapid evaluation technique. The capacity of the building for an initial major yielding condition and for an ultimate load condition are estimated. By use of the capacity spectrum method, the capacity curve is reconciled with the demand curve of the EQ-II response spectrum. An example of the procedure are given in sheets 6 through 12. The results of the evaluation of all the structures are summarized in a report that includes capacities, percent damage, and damage costs.

Sheet 3 of 12

Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 3 of 12).

PRELIMINARY SCREENING

(PRE INSPECTION DATA)

BUILDING NO. 55 INSPECTED BY SAF DATE 1/15/86

DESCRIPTIVE TITLE (Current Use) HOSPITAL BUILDING

CLASSIFICATION ESSENTIAL

AVAILABILITY OF DESIGN DATA DRAWINGS AND CALCULATIONS ARE AVAILABLE

BUILDING DATA:

Number of Stories 3

Height 35' Plan (Show Dimensions) 48' x 192'

CONSTRUCTION:

Structural System Structural Steel Frame

Roof METAL DECK WITH LIGHTWEIGHT FILL

Intermediate Floors METAL DECK WITH CONC. FILL

Ground Floors SLAB ON GRADE

Foundations

Interior Walls

Exterior Walls

LATERAL FORCE RESISTING SYSTEM DMRSF TRANSV. BRACED FRAME LONGIT.

EVALUATION:

General Condition

Earthquake Damage Potential

DAMAGE OBSERVED:

COMMENTS:

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Figure F–1. Sample screening and evaluation of a large military installation. (Sheet 4 of 12)

PRELIMINARY SCREENING

BUILDING NO. 105 INSPECTED BY SAF DATE 1/17/86

DESCRIPTIVE TITLE (Current Use) COMMUNICATIONS BUILDING
Built circa 1910

CLASSIFICATION ESSENTIAL

AVAILABILITY OF DESIGN DATA AS-BUILT DRAWINGS AVAILABLE

BUILDING DATA:

Number of Stories 3

Height 52'-6" Plan (Show Dimensions) 62' x 186'

CONSTRUCTION:

Structural System REINFORCED CONCRETE FRAME

Roof 6" R/C SLAB AND BEAMS

Intermediate Floors SAME

Ground Floors SLAB ON GRADE

Foundations R/C COLUMN FOOTINGS AND WALL FOOTINGS

Interior Walls NONE

Exterior Walls BRICK WALLS

LATERAL FORCE RESISTING SYSTEM UNREINFORCED BRICK MASONRY PARTIALLY CONFINED BY R/C FRAMES

EVALUATION:

General Condition GOOD

Earthquake Damage Potential HIGH

DAMAGE OBSERVED: NONE OBSERVED

COMMENTS:

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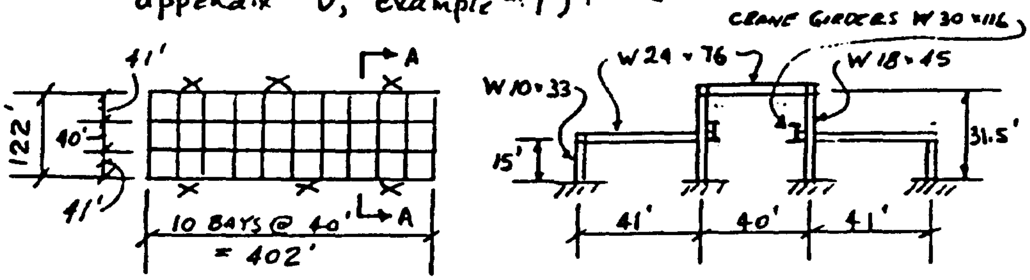
Figure F–1. Sample screening and evaluation of a large military installation. (Sheet 5 of 12)

Building 131 - Pipe and Copper Shop (Circa 1940)

PRELIMINARY EVALUATION (para 4-2)

FIND: Capacity of the structure (para 4-2c) using a rapid evaluation technique.

GIVEN: One story steel frame building with a high crane bay (Note: Same building described in appendix D, example #1).



PLAN

SECTION A

TRANSVERSE DIRECTION:

Steel moment frames: Girders . W 24 x ~~74~~ 76
Interior Col. W 18 x 45
Exterior Col. W 10 x 33

Crane girders W 30 x 116
Column base plate fixed with 4-1½" bolts

LONGITUDINAL DIRECTION:

Diagonal steel bracing: 3-bays on each side
∠ 3½ x 3 x 5/16
with 4-⅞" ϕ rivets ea. end, single shear

Note; upper portion of high bay also braced.

WEIGHTS: Roof and framing 25 psf
Walls (equiv) 10
Misc 5
} 40 psf x 122' x 402' = 1962 K

ONE 40' BAY: W_B = 195 K

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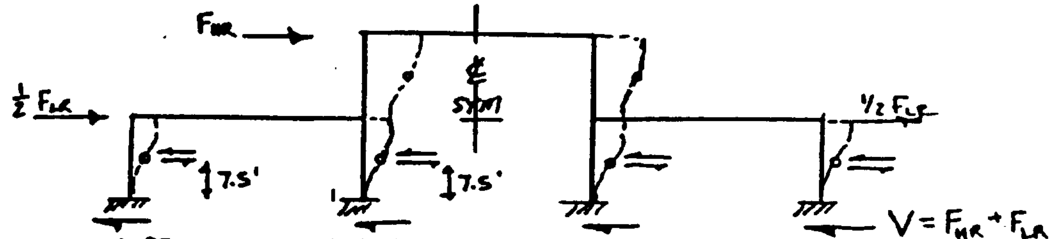
Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 6 of 12)

Building 131 (CONTINUED)

CAPACITY IN TRANSVERSE DIRECTION

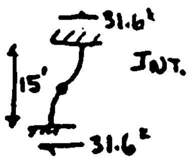


Assume inflection point at midheight
- GIRDER REL. RIGID
- BASE FIXED

COL.	W10×33	W18×45	
I	171 in⁴	706 in⁴	
S	35 in³	79 in³	
Z	39 in³	90 in³	
Rel. k $\left(\frac{I}{L^3}\right)$	0.10	0.40	
M_Y	105 k-i	237 k-i	Moment at first yield = 36 ksi × S ÷ 12 (k-i)
M_P	117 k-i	270 k-i	36 ksi × Z ÷ 12 - PLASTIC MOMENT
M_u	146 k-i	338 k-i	1.25 × M_P - ULTIMATE AT 125% of plastic
V_Y	14 k	31.6 k	$V = \frac{M}{7.5'}$ at assumed inflection point.
V_P	15.6 k	36.0	
V_u	19.5 k	45.1	

First Yield	7.9 k	31.6 k	Interior Col. yields, exterior at $\frac{0.10}{0.40}$ $\Sigma V_1 = 2(7.9 + 31.6) = \underline{79}$ k
2ND Yield	14.0 k	36.0 k	Interior at M_p, exterior at M_Y $\Sigma V_2 = 2(14 + 36) = \underline{100}$ k
ULT.	19.5 k	45.1 k	$\Sigma V_u = 2(19.5 + 41.1) = \underline{121}$ k

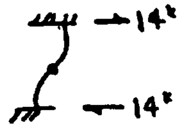
Deflection at first YIELD



INT. COL I = 706

$$\Delta = \frac{PL^3}{12EI} = \frac{31.6 \times 15^3 \times 12^3}{12 \times 29 \times 10^3 \times 706} = \underline{\underline{0.75''}}$$

Defl. when ext. col. yields



$$\Delta = \frac{14.0 \times 15^3 \times 12^3}{12 \times 29 \times 10^3 \times 171} = \underline{\underline{1.37''}}$$

Assumes Rigid Girder. Increase Values for joint rotation. Assume 20% increase.

Deflections at low roof; w/ est. for joint rotation $\left\{ \begin{array}{l} \Delta_y \approx 0.75 \times 1.2 = 0.9'' \\ \Delta_{y2} \approx 1.37 \times 1.2 = \underline{\underline{1.6}}'' \end{array} \right.$

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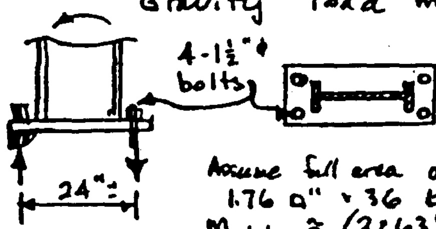
Figure F–1. Sample screening and evaluation of a large military installation. (Sheet 7 of 12)

Building 131, Capacity Transverse Dir. (cont.)

High bay: $F_{HR} \simeq F_{LR}$ (twice acceleration, ½ mass)
$\therefore V_{HR} \simeq \frac{1}{2} V$, OK by inspection

Check Following:
- Base plate capacity
- Girder capacities & Connections
- Gravity load moments

• Base plate



Assume full area at 36 ksi
$1.76\ \square'' \times 36\ ksi = 63^{k}/bolt$
$M_{plate} \simeq (2 \times 63^{k}) \times 2' = \underline{252}^{k\text{-}'} > 237^{k\text{-}'}$

Bolts should hold until column yields
Conc. Foundation OK for loads.

• Girders : W 24×76 : $I = 2100$, $S = 176$

$M_y = 36 \times 176 \div 12 = 528^{k\text{-}'}$ > Col. Capacities.
Beam - Column conn. OK by inspection

• Gravity loads

At 25 psf × 40' trib. width = $1000^{\#}/' = 1^{k}/'$
FEM at $wl^2/12 = 1 \times 40^2/12 = 133^{k\text{-}'}$
Moment will reduce due to moment distribution
ESTIMATE $M_{DL} \simeq 90^{k\text{-}'}$
Beam: $M_y - M_{DL} = 528 - 90 = 438^{k\text{-}'} >$ Col.
Col.: Gravity moments will add to some columns and subtract from others - Capacities balance out.

TRANSVERSE CAPACITY SUMMARY ($W = 195^{k}$/BAY)

	V	C_B (V/W)	Δ_{LR}	Δ_{HR} ($2 \times \Delta_{LR}$)	PFR (EST)	α (EST)	S_d $\Delta_{HR} \div PFR$	S_a $C_B \div \alpha$	T $2\pi\sqrt{\frac{S_d}{S_a g}}$
FIRST YIELD	79	0.41	0.90"	1.8"	1.25	0.90	1.4"	0.46 g	0.56 s.
2ND YIELD	100	0.51	1.6"	3.2"	↓	↓	2.6"	0.57 g	0.68 s.
ULT w/. Δ=0.015H	121	0.62		5.7"			4.6"	0.69 g	0.82 s.

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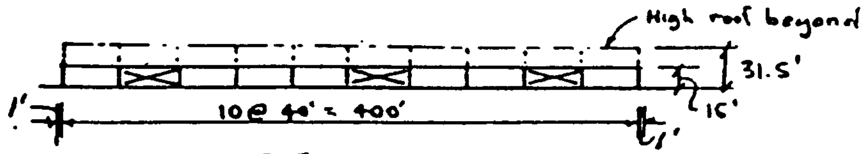
Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 8 of 12)

Building 131 (CONT.)

CAPACITY IN LONGITUDINAL DIRECTION



ELEVATION

Assumptions based on field inspection:

1. High roof braced to low roof to transfer lateral forces
2. Low roof system horizontally braced to transfer from interior column lines to exterior columns
3. Interior longitudinal frame braced at upper level and by crane girder to develop frame action in W18×45 columns
4. Exterior columns fixed at top and base
5. Exterior facade negligible for lateral resistance.

Lateral force resistance:

CASE I All forces taken by 6 sets of X-braces (3 sets each side of building)

CASE II ~~Compression braces buckle~~, some ~~participation by longitudinal~~ frames { VOID, NO COMPRESSION CAPACITY

CASE III Diagonal braces fail, resistance by longitudinal frames. (weak direction of columns).

DIAGONAL BRACES ∠ 3 1/2 × 3 × 5/16

$L = \sqrt{15^2 + 40^2} = 42.7$ feet ; Joined at center $L_x = 21.3'$

$r_z = 0.63''$, $r_x = 1.06$, $r_y = 0.909$, $A = 1.93 \square''$

For r_z, $L = 21.3 - 2' = 19.3$; $L/r = 367$ NG no axial comp.

No COMPRESSION CAPACITY

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Figure F–1. Sample screening and evaluation of a large military installation. (Sheet 9 of 12)

Building 131, Capacity Long. Dir. (cont)

CASES I & II   Braces effective in tension only

$\longrightarrow F_D = \frac{40}{42.72} \times 36\text{ ksi} \times 1.93\,\square'' = \underline{\underline{65^k}}$

CHECK CONNECTIONS   4 - 7/8" rivets (Assume A 502-1)

AISC table, Single shear allowable = 9.02 k/rivet

YIELD CAPACITY ≈ 1.7 × ALLOWABLE = 15.3 k/rivet

$F_{D(conn)} = \frac{40}{42.72} \times 4 \times 15.3 = \underline{\underline{57.5}} < 65$

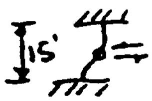
Conn. Governs

$V_{yield} = 6 \text{ braces} \times 57.5^k = \underline{\underline{345}}^k$

CASE III   Column bending action

2 × 11 - W 10 × 33 $I_{yy} = 36.5\,in^4$; $S_{yy} = 9.16\,in^3$ } Close
2 × 11 - W 18 × 45 $= 34.8\,in^4$ $= 9.32\,in^3$ } use avg.

Use 44 col with $I = 35.6\,in^4$ and $S = 9.24\,in^3$

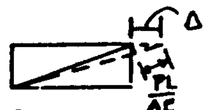


$V_{col} = \frac{M}{7.5'} = \frac{36\text{ ksi} \times S}{7.5 \times 12} = \frac{36 \times 9.24}{7.5 \times 12} = \underline{3.7^k/col}$

V_{yield} for 44 columns $= 44 \times 3.7 = \underline{\underline{163}}^k$

DEFLECTIONS

BRACE:



$\frac{PL}{AE} = \frac{(4 \times 15.3)\,42.7 \times 12}{1.93 \times 29 \times 10^3} = \underline{0.56}''$

$\Delta_B = \left[(42.720' + .56/12)^2 - 15^2\right] - 40' = 0.050' = \underline{\underline{0.60}}''$ CASE I/II

Columns: $\Delta_C = \frac{PL^3}{12EI} = \frac{3.7 \times 15^3 \times 12^3}{12 \times 29000 \times 35.6} = \underline{\underline{1.74}}''$ CASE III

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Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 10 of 12)

Building 131, Capacity Long. Dir (CONT.)

Relative rigidities - Braces and Columns

BRACES: $\frac{345^k}{0.60''} = 575$ k/in
COLUMNS: $163^k/1.74'' = 94$ k/in
} ratio = 6.1 to 1

First Yield When braces are taking 345^k at $\Delta = 0.60''$

Columns are resisting $\frac{0.60}{1.74} \times 163^k = 56^k$ at $\Delta = 0.60''$

Total Shear, $V_y = 401^k$ @ $\Delta = 0.6''$ L.R

Braces Fail Assume 1/3 fail, remainder incr. capacity by 15%

Braces: $2/3\,(345)\,1.15 = 264^k$ @ $\Delta = \frac{3}{2}\,0.6 = 0.9''$

Columns $(0.90/1.74)\,163 \quad 84^k$

$V_{u\text{-}1} = 348^k$ @ $\Delta = 0.9''$ L.R

ULTIMATE ALL BRACES FAIL

Columns Yield at 163^k @ $\Delta_y = 1.74''$

$\Delta_{max} = 0.015 \times H = 0.015 \times 15 \times 12 = 2.7''$ L.R
$\Delta_{max}/\Delta_y = 2.7/1.74 = 1.55$
Assume 5% Capacity increase: $163^k \times 1.05 = 171^k$ @ $2.7''$

LONGITUDINAL CAPACITY SUMMARY ($W = 1962^k$)

	V	C_B (V/W)	Δ_{LR}	Δ_{HR} $2 \times \Delta_{LR}$	PFR (est)	α (est)	S_d $\Delta_{HR} \div PFR$	S_a $C_B \div \alpha$	T $2\pi\sqrt{\frac{S_d}{S_a g}}$
First Yield	401^k	0.204	0.6"	1.2	1.25	0.90	0.96"	0.23 g	0.65 s.
First Fail.	348^k	0.177	0.9	1.8	1.25	0.90	1.44"	0.20 g	0.85 s.
Ult @ $\Delta = 0.015H$	171^k	0.087	2.7	5.4	1.25	0.90	4.32"	0.097 g	2.13 s

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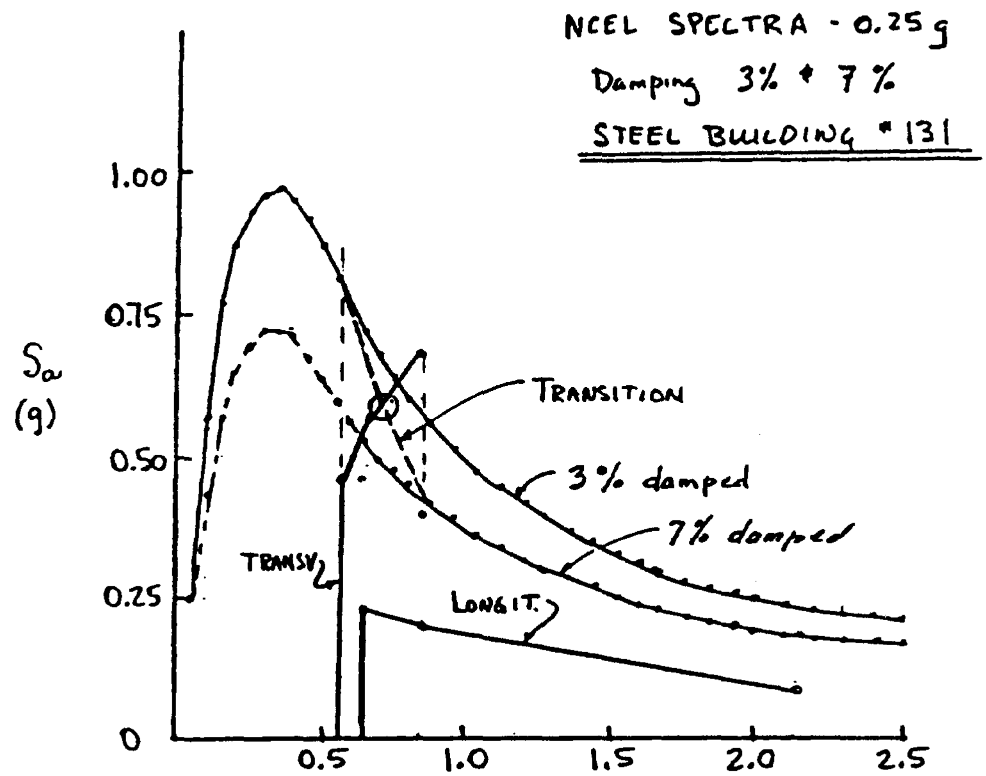
Figure F-1. Sample screening and evaluation of a large military installation. (Sheet 11 of 12)

CAPACITY SPECTRA		TRANSVERSE			LONGITUDINAL		
	S_a	0.46	0.57	0.69	0.23	0.20	0.097
	T	0.56	0.68	0.82	0.65	0.85	2.13



Damage, percent

MAX G	TRANSV	LONGIT.	COMBINED
0.25	52%	>100%	84
0.125	0	50% *	33
0.063	0	0	0

* Capacity curve follows demand curve, assume 50%.

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Figure F–1. Sample screening and evaluation of a large military installation. (Sheet 12 of 12)

DESIGN EXAMPLE F-2

BRICK BUILDING WITH CONCRETE FRAMING SYSTEM

Description of Structure. A 3-story communications building built circa 1905 with vertical load carrying concrete frames and exterior unreinforced brick masonry walls. The floor and roof construction is comprised of reinforced concrete slabs and beams. The building is supported on concrete pile caps and timber piles. The structural design concepts are illustrated on sheets 2, 3, and 4.

Construction Outline.

Roof:
- Built-up roofing.
- Reinforced concrete slabs, beams, and girders.

2nd and 3rd Floors:
- Reinforced concrete slabs, beams, and girders.

1st Floor:
- Reinforced concrete slab-on-grade.

Foundation:
- Reinforced concrete tie beams and pile caps supported on timber piles.

Exterior Walls:
- Unreinforced brick masonry with terra cotta facade.

Partitions:
- Clay tile walls and wood stud walls with gypsum board sheathing.

Background. As a result of the inventory reduction and preliminary screening process the building was included in the list of buildings requiring a preliminary evaluation. On the basis of the preliminary evaluation (sheets 6, 8, and 9), upgrading concepts will be developed. A summary of the Acceptance Criteria and the determination of the Site Response Spectra are shown in the sheets 5, 6, and 7.

Sheet 1 of 25

Figure F-2. Brick building with concrete framing system. (Sheet 1 of 25)

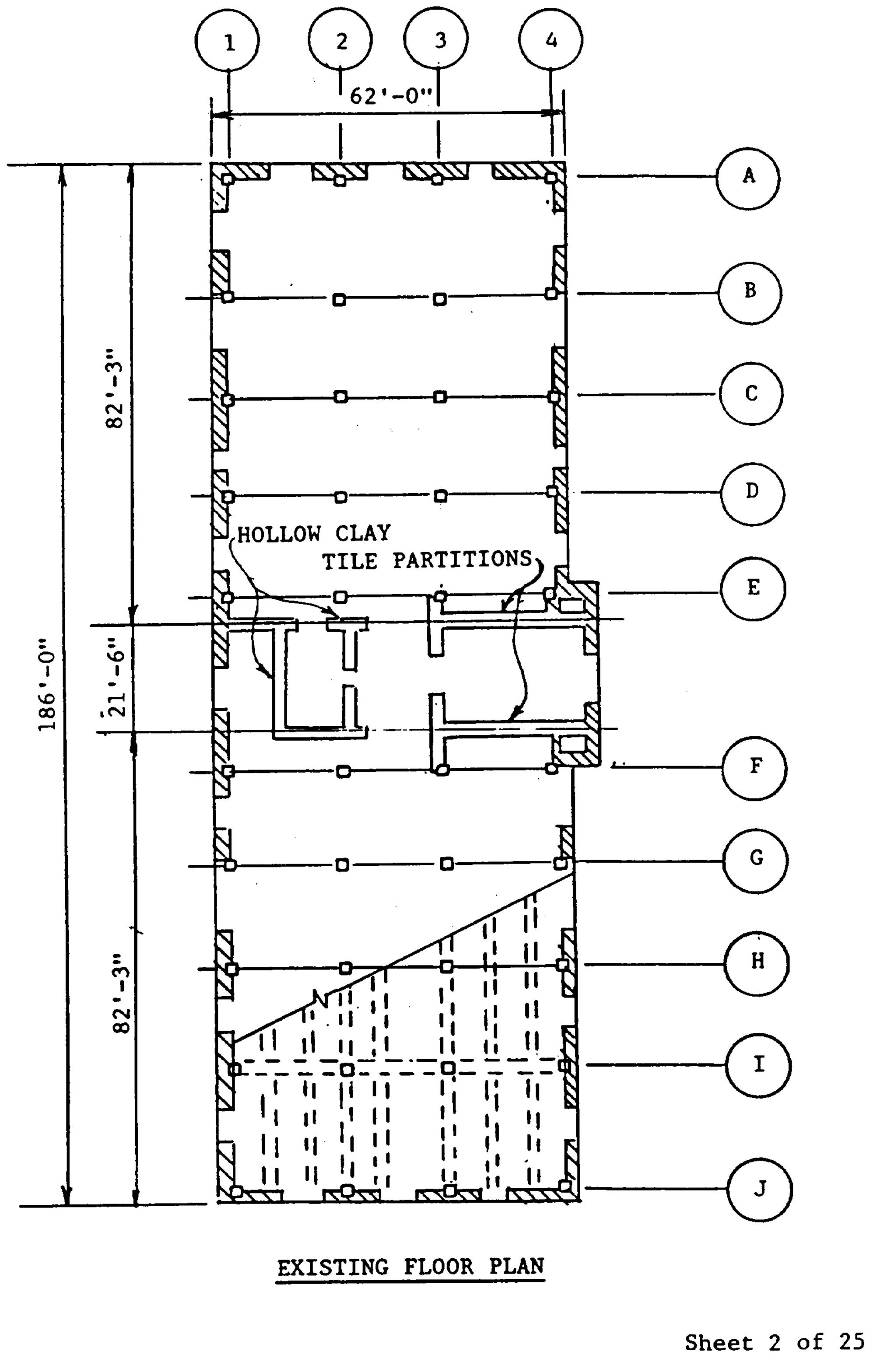


Figure F-2. Brick building with concrete framing system. (Sheet 2 of 25)

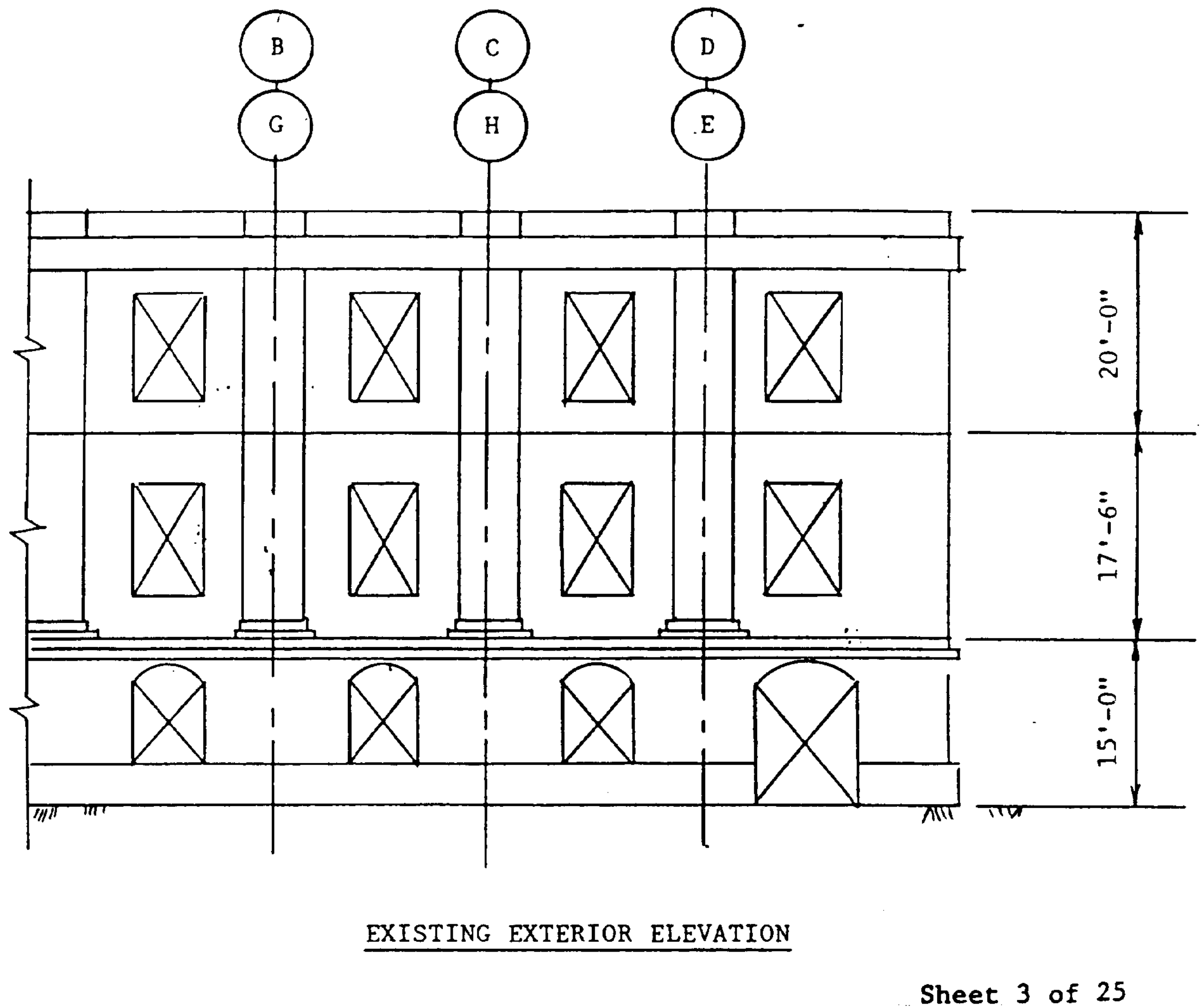
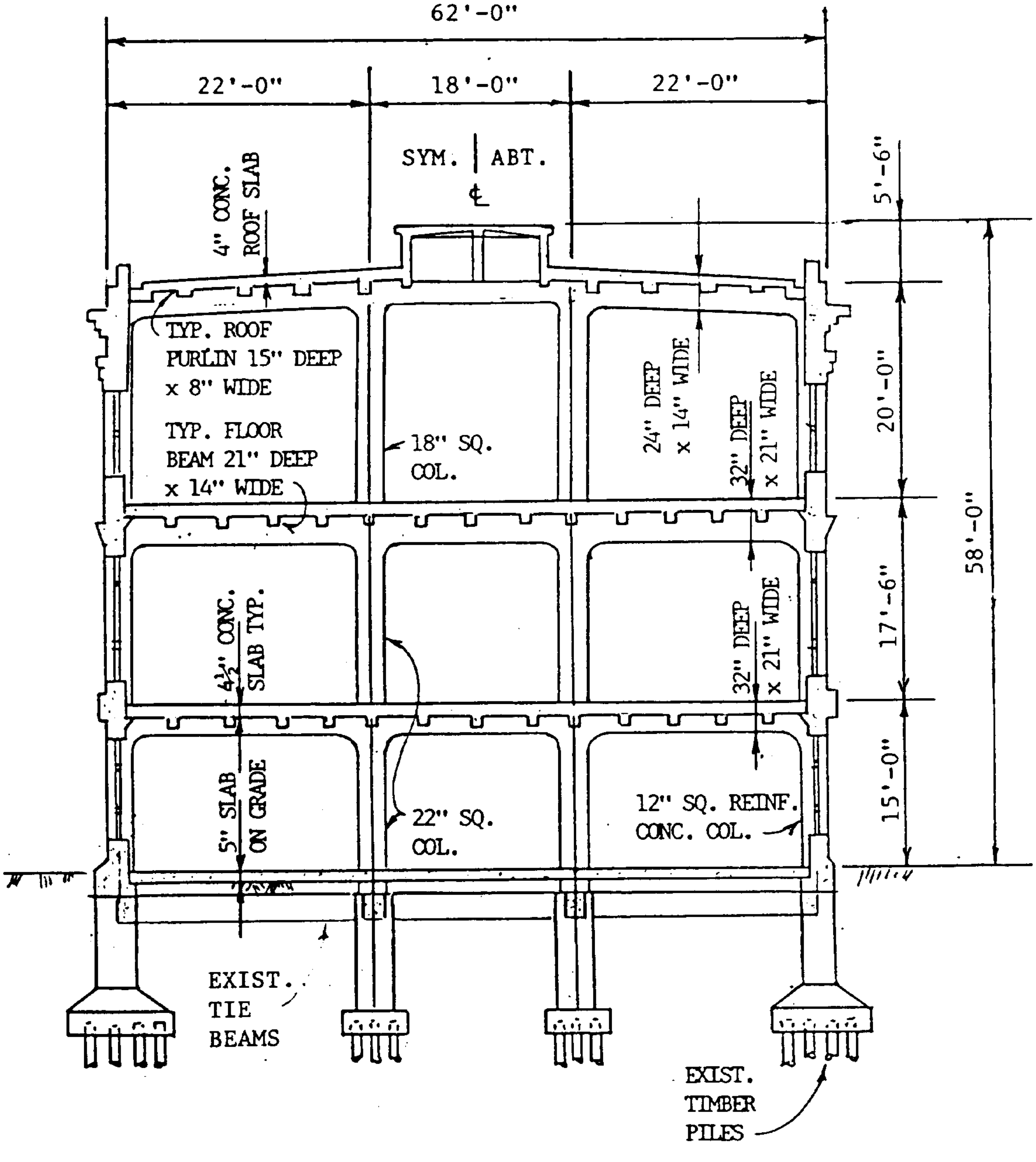


Figure F–2. Brick building with concrete framing system. (Sheet 3 of 25)



TYPICAL TRANSVERSE BUILDING SECTION

Sheet 4 of 25

Figure F–2. Brick building with concrete framing system. (Sheet 4 of 25)

The Acceptance Criteria for the seismic resistance is that presented for the post yield analysis for EQ-II, Method 1 (refer to SDG paras 4-4 and 5-5).

Classification:	Essential
Loading Combination:	DL + 0.25LL + EQ
Ultimate Strength Capacities:	ACI 318 Strength Design
Inelastic Demand Ratios:	
Nonductile Conc. Frames	
Columns	1.00
Beams	1.25
Reinf. Conc. Shear Walls	
Single Curtain Reinf.	Shear-1.10, Flexure-1.5
Double Curtain Reinf.	Shear-1.25, Flexure-2.0
Material Properties:	
Concrete	f'_c = 4000 psi(New)
	f'_c = 3000 psi(Exist)
Reinforcement	F_y = 60 ksi(New)
	F_y = 33 ksi(Exist)
Unreinf. brick masonry	E_m = 1000 ksi(Exist)
Story Drift Limitation:	0.006 x Story Height

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Figure F-2. Brick building with concrete framing system. (Sheet 5 of 25)

Site Response Spectra. The site response spectra are developed in accordance with the procedure in Chapter 3 of the SDG:

Building Classification: Essential Facility

Ground Motion Spectra: ATC 3-06 Map Contour Level, $A_a = A_v = 0.10$

Soil Classification: $S_i = 1.5$ (Type S3)

Earthquake I
- Damping = 5%, D.F. = 1.00 (SDG table 3-7)
- $A_a = A_v = 0.04g$ (Design Ground Motion, SDG table 3-4)
- $S_a = D.F.\ (1.22A_vS_i)/T = 0.073g/T$ less than $D.F.(2.5)A_a = 0.10g$ max

Earthquake II
- Damping = 10%, D.F. = 0.80
- $A_a = A_v = 0.12g$
- $S_a = D.F.\ (1.22A_vS_i)/T = 0.176g/T$ less than $D.F.(2.5)A_a = 0.24g$ max

EQ-II/EQ-I = 0.24/0.10 = 2.40

The resulting spectra are shown on sheet 7.

Preliminary Evaluation. The lateral force resisting system primarily consists of unreinforced brick masonry piers and walls, partially confined by the concrete frames. The concrete frames are capable of only a minimal amount of lateral force resistance as there is very little continuity in the reinforcement at the column-beam joints. Since the existing walls would be required to resist most of the seismic force by relative rigidity, the existing concrete frames will be ignored in the preliminary evaluation. A rapid approximation of the seismic demand is made by assuming that the demand spectral acceleration (S_a) for the first mode is 0.24g (i.e., T less than 0.7 sec) and that the base shear coefficient (C_B) = $0.86S_a$ = 0.21g (α = 0.86 per SDG para 5-3a(2)(c)). The seismic forces will be distributed to the various stories in accordance with the static design provisions of the BDM. The capacity of the existing structure will be approximated by calculating the average shear stress (story shear divided by the total net wall area in each direction) for each story. See sheets 8 and 9 for this preliminary evaluation. The structural deficiencies identified were the non-conforming moment-resisting concrete frames and the unreinforced brick masonry walls in both shear and flexure. Results show conclusively that the building does not satisfy the acceptance criteria. Therefore, a detailed structural analysis of the existing as-is building is not required. Upgrading concepts will be developed and the acceptance criteria of the upgraded structure will be confirmed by a detailed structural analysis.

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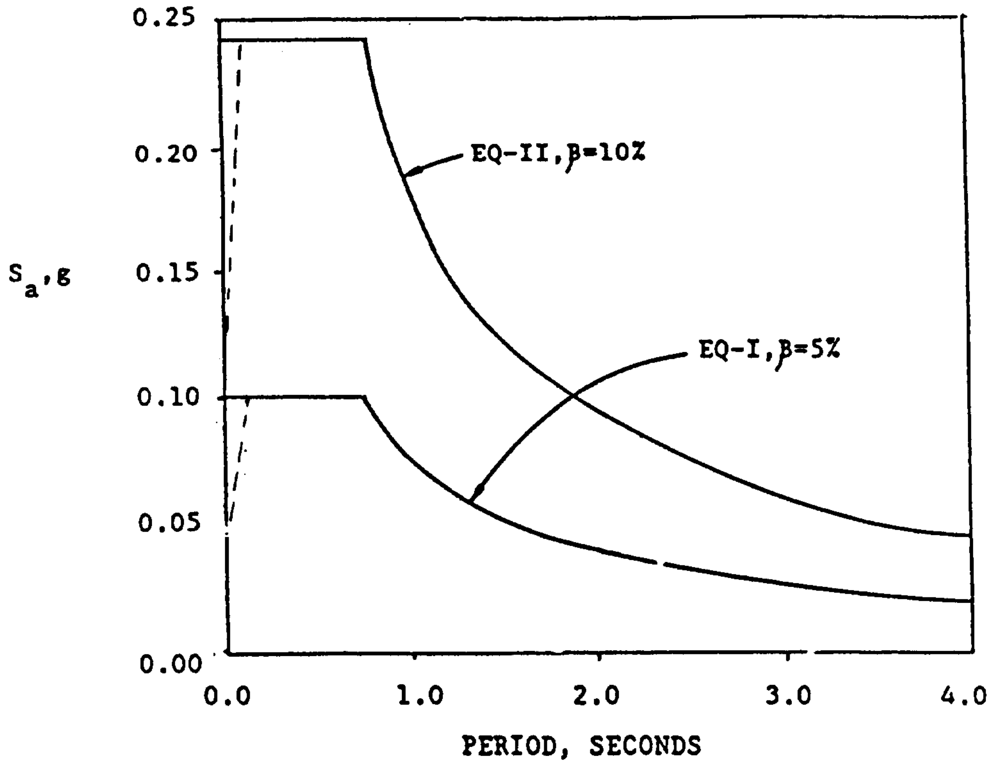
Figure F-2. Brick building with concrete framing system. (Sheet 6 of 25)

DESIGN RESPONSE SPECTRA FOR EQ-I AND EQ-II



EQ	β		PERIOD							
			0.0	.730	1.0	1.25	1.5	2.0	3.0	4.0
I	5%	S_a, g	.04	.10	.073	.058	.049	.037	.024	.018
II	10%	S_a, g	.12	.24	.176	.141	.117	.088	.058	.044
		S_d, in	0	1.25	1.72	2.16	2.58	3.45	5.11	6.89

* SPECTRAL DISPLACEMENT $S_d = S_a (T/2\pi)^2 g$

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Figure F-2. Brick building with concrete framing system. (Sheet 7 of 25)

RAPID EVALUATION

Longitudinal Direction

Lateral force resisted by two exterior walls.

Total length of walls = 2 x 186' = 372'.

Windows reduce effective length by 1/3.

Effective Length = 2/3 x 372' = 248'

Assume 18" brick wall at 15 psi shear strength
at 50 psi shear ultimate
(to be confirmed by tests)

At 15 psi V = 248' x 12 x 18" x 0.015 = 803^k
At 50 psi V = 248' x 12 x 18" x 0.050 = 2680^k

Calculated Weight: $10,000^k$

Equivalent to 17#/cu ft
or 290#/sq. ft

Estimate Capacity

$$C_B \text{ - yield?} \simeq \frac{803}{10,000} = \underline{0.08}g$$

$$\text{- ultimate} \simeq \frac{2680}{10,000} = \underline{0.27}g$$

NOTE: Typical pier width/height ratio = 1.20, therefore assume shear governs.

DEMAND of earthquake: $S_a = 0.24g$ (Sheet 6)
$C_B \simeq 0.86\ S_a = 0.21g$

Requires about 40 psi capacity (e.g., (0.21g/0.27g)x50).

If strength is confirmed by tests, the longitudinal direction could work if connections are acceptable.

NOW CHECK THE TRANSVERSE DIRECTION

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Figure F-2. Brick building with concrete framing system. (Sheet 8 of 25)

RAPID EVALUATION (continued)

Transverse Direction

Lateral Force Resistance

Two exterior brick walls: 44' x 2 = 88' effective length

V_{ext} = 88' x 12 x 18" x 50 psi = 950^k

Two interior hollow clay tile

V_{int} = 2 x 50'± x 12 x 8" x 20 psi = 192^k

Total resistance ≃ $1{,}142^{kips}$

Estimated capacity: $V/W = \frac{1142}{10{,}000} = 0.114$

is less than the demand C_B = 0.21 (Sheet 8).

Conclude: Weak in Transverse Direction

Even with liberal allowances for material strength, resistance about 1/2 demand.

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Figure F-2. Brick building with concrete framing system. (Sheet 9 of 25)

Development of Seismic Upgrade.

Structural Upgrading Concepts. Three concepts were considered:

1. Install reinforced concrete gunite against the interior faces of the exterior unreinforced brick walls and add new interior cast-in-place reinforced concrete walls.

2. Install vertical structural steel plate panels as infill walls within existing transverse interior concrete framing and use diagonal steel braces for the longitudinal direction.

3. Construct exterior buttresses to give lateral support to the existing building.

No. 1, above, was selected as the recommended concept. Plans, elevations, and details are shown on sheets 11 through 17 and a discussion on the analysis is contained on sheet 18. For concept No. 2, steel walls and bracing, it was considered to be difficult to obtain satisfactory connections between steel and concrete because of the high force levels. For concept No. 3, exterior buttresses, a preliminary cost comparison indicated that it would not be as cost effective as concept No. 1. Also, concept No. 3 would distract from the historic significance of the building. A disadvantage of concept No. 1 was the blocking out of existing windows; however, it was determined that this would not be detrimental to the planned use of the building. It should be noted that, if it were mandatory to minimize on-going operations in this building, then the additional costs of concepts Nos. 2 or 3 might be justified.

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Figure F-2. Brick building with concrete framing system. (Sheet 10 of 25)

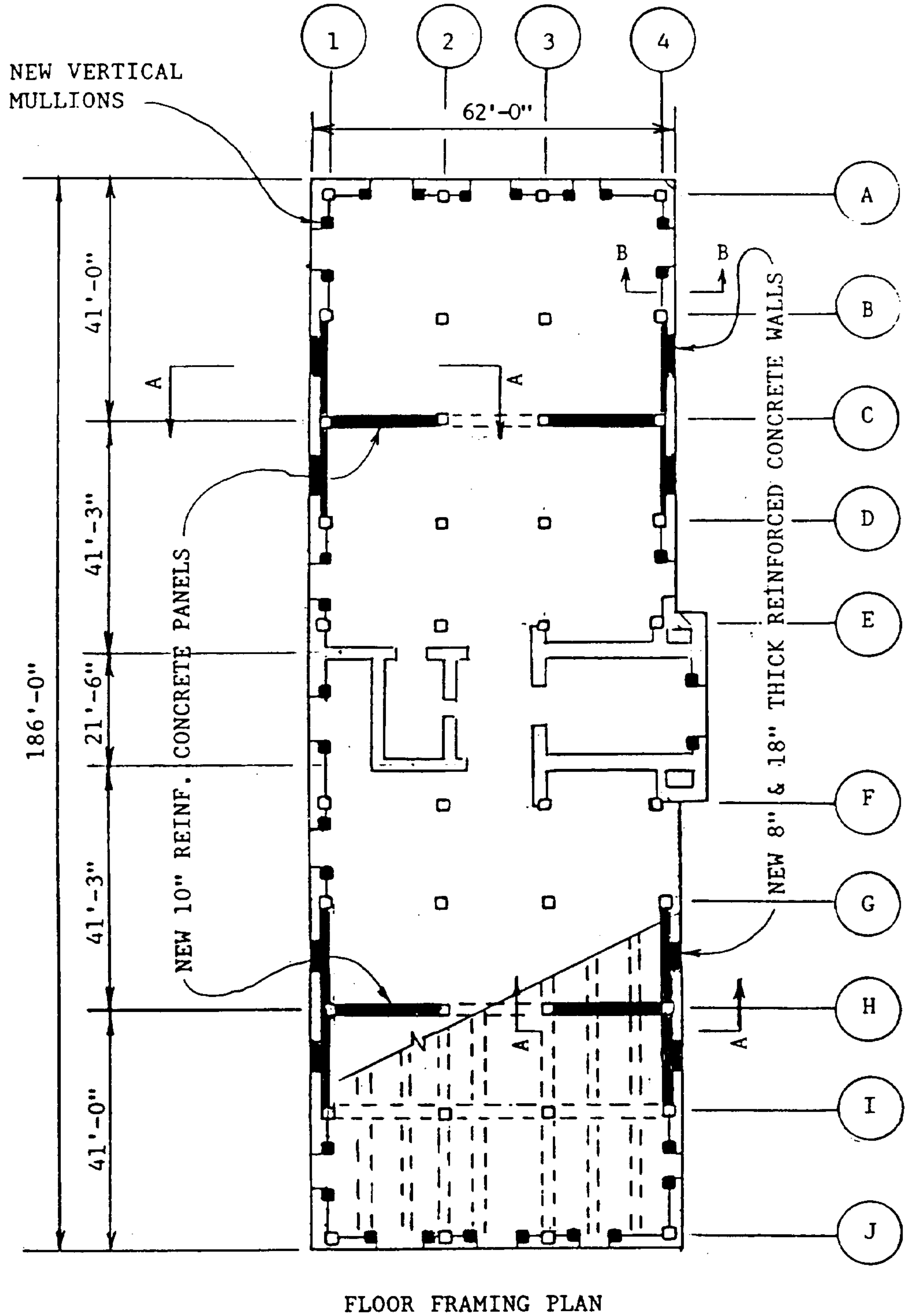

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Figure F-2. Brick building with concrete framing system. (Sheet 11 of 25)

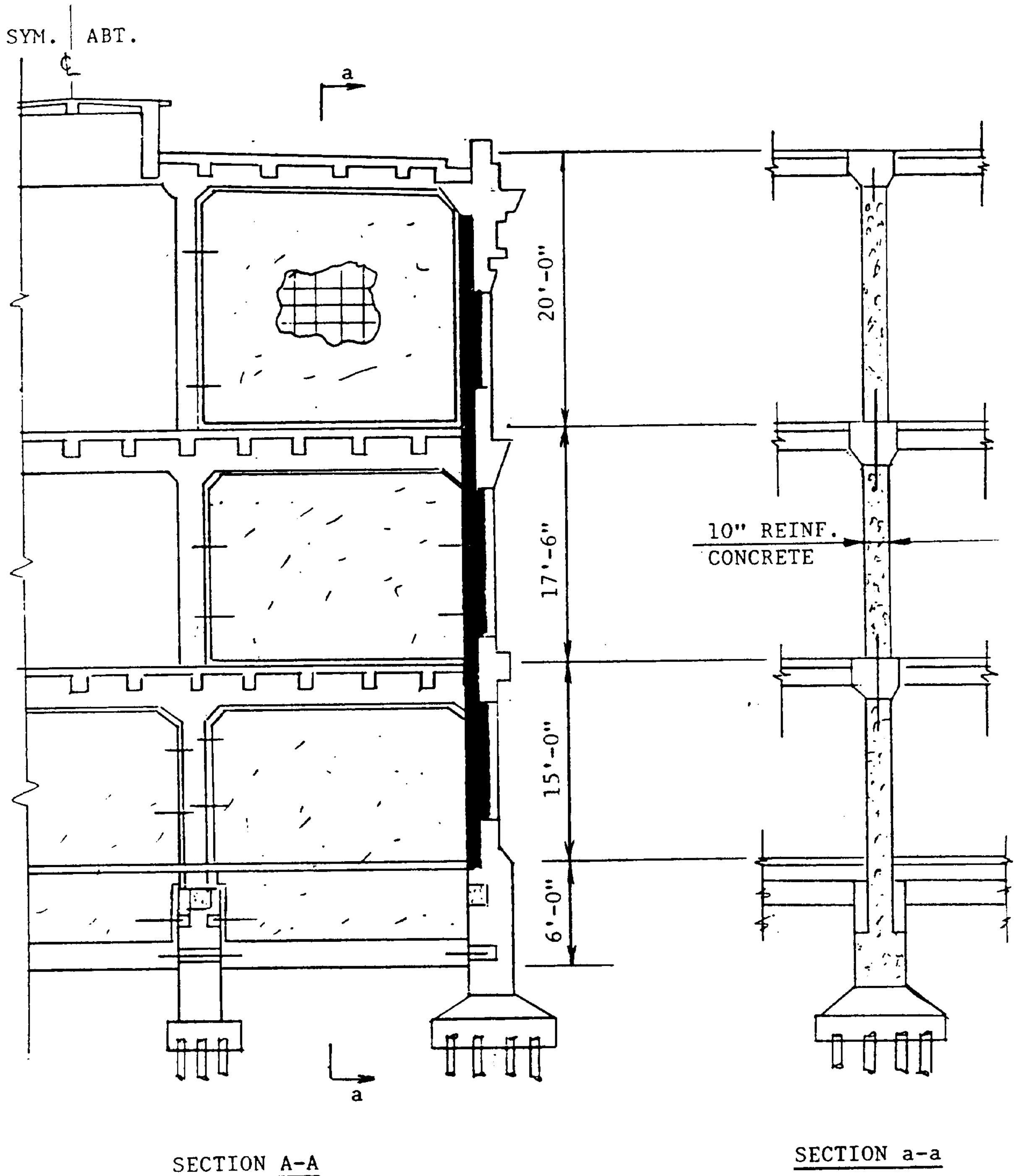

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Figure F–2. Brick building with concrete framing system. (Sheet 12 of 25)

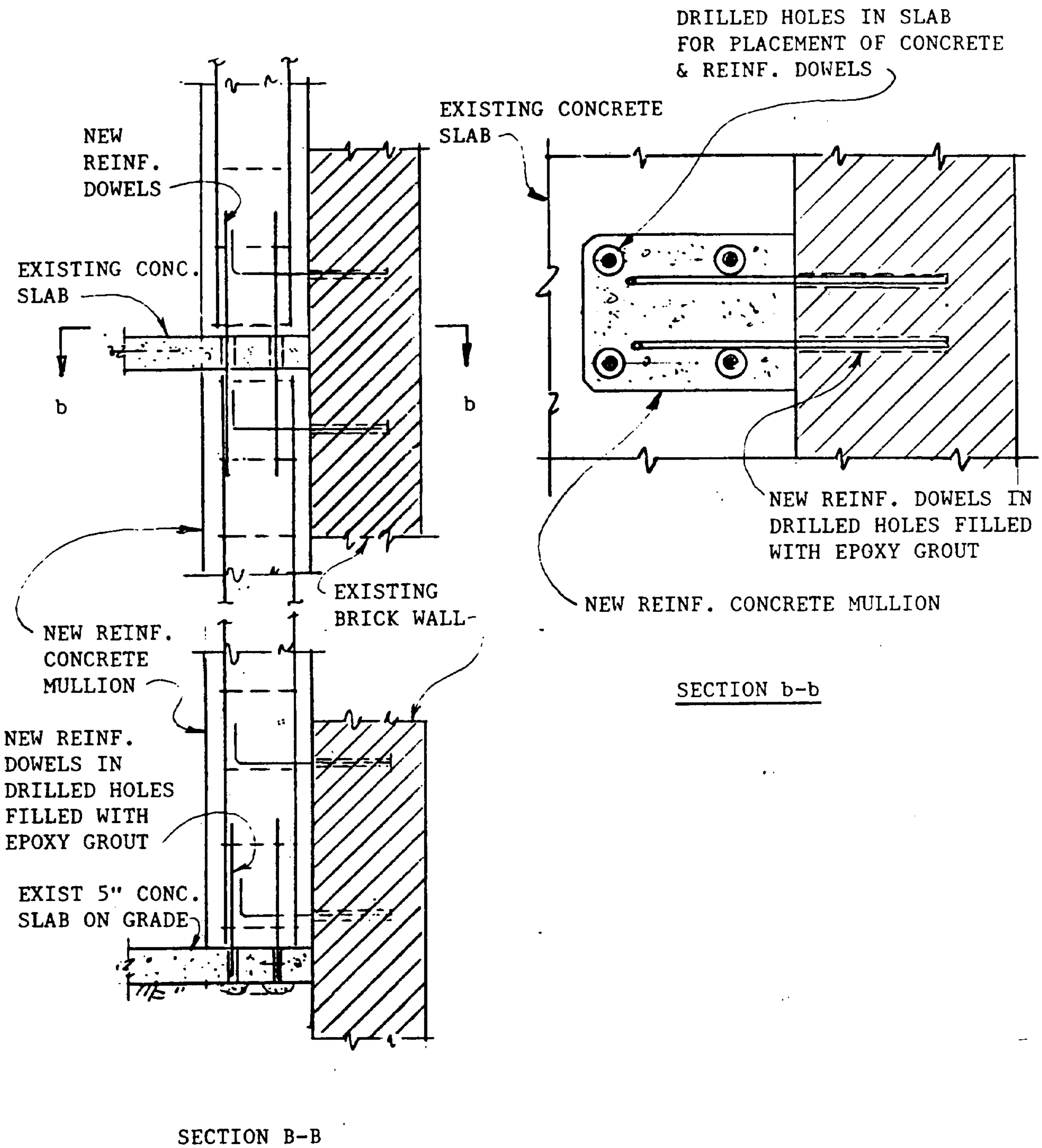

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Figure F-2. Brick building with concrete framing system. (Sheet 13 of 25)

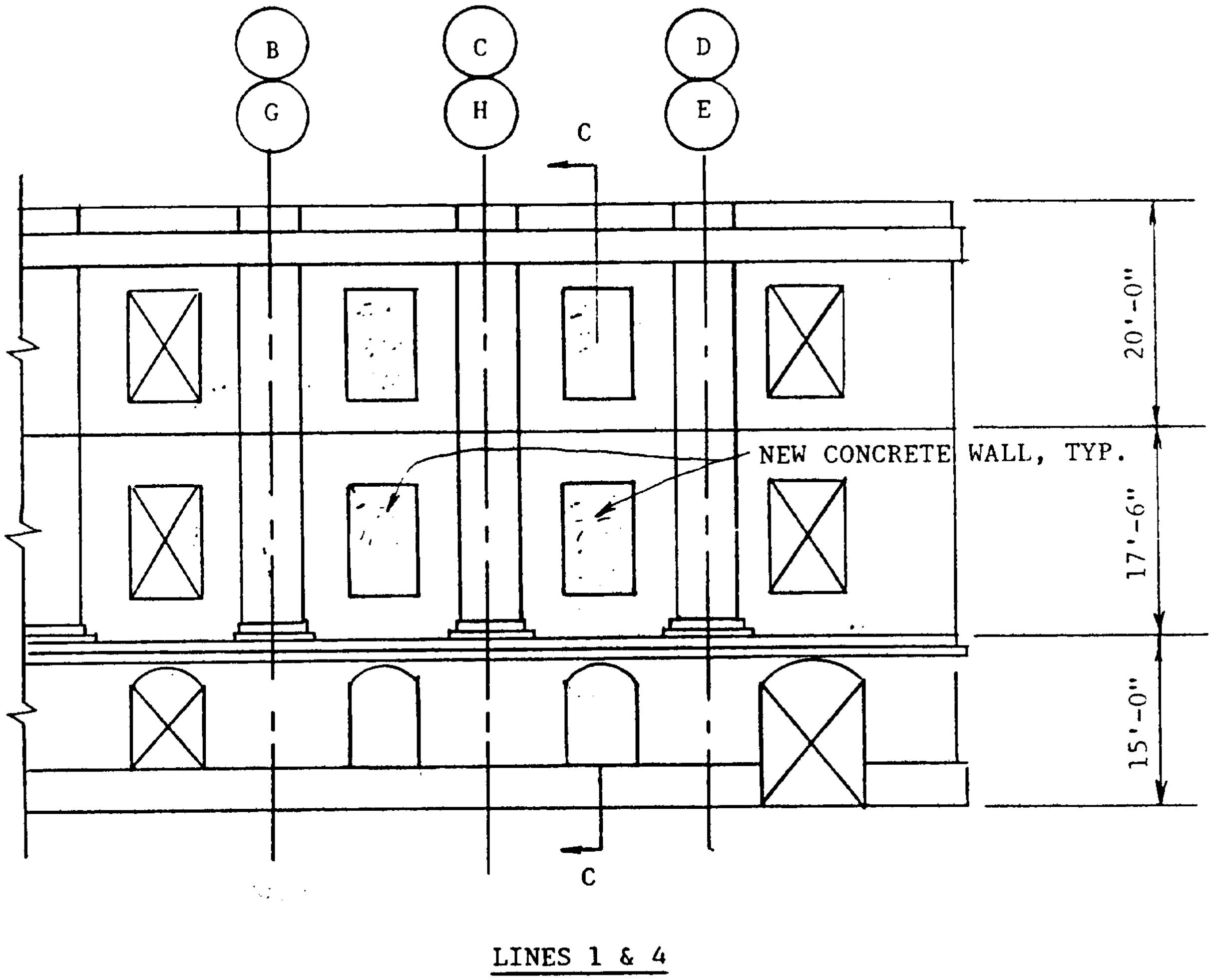

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Figure F-2. Brick building with concrete framing system (Sheet 14 of 25)

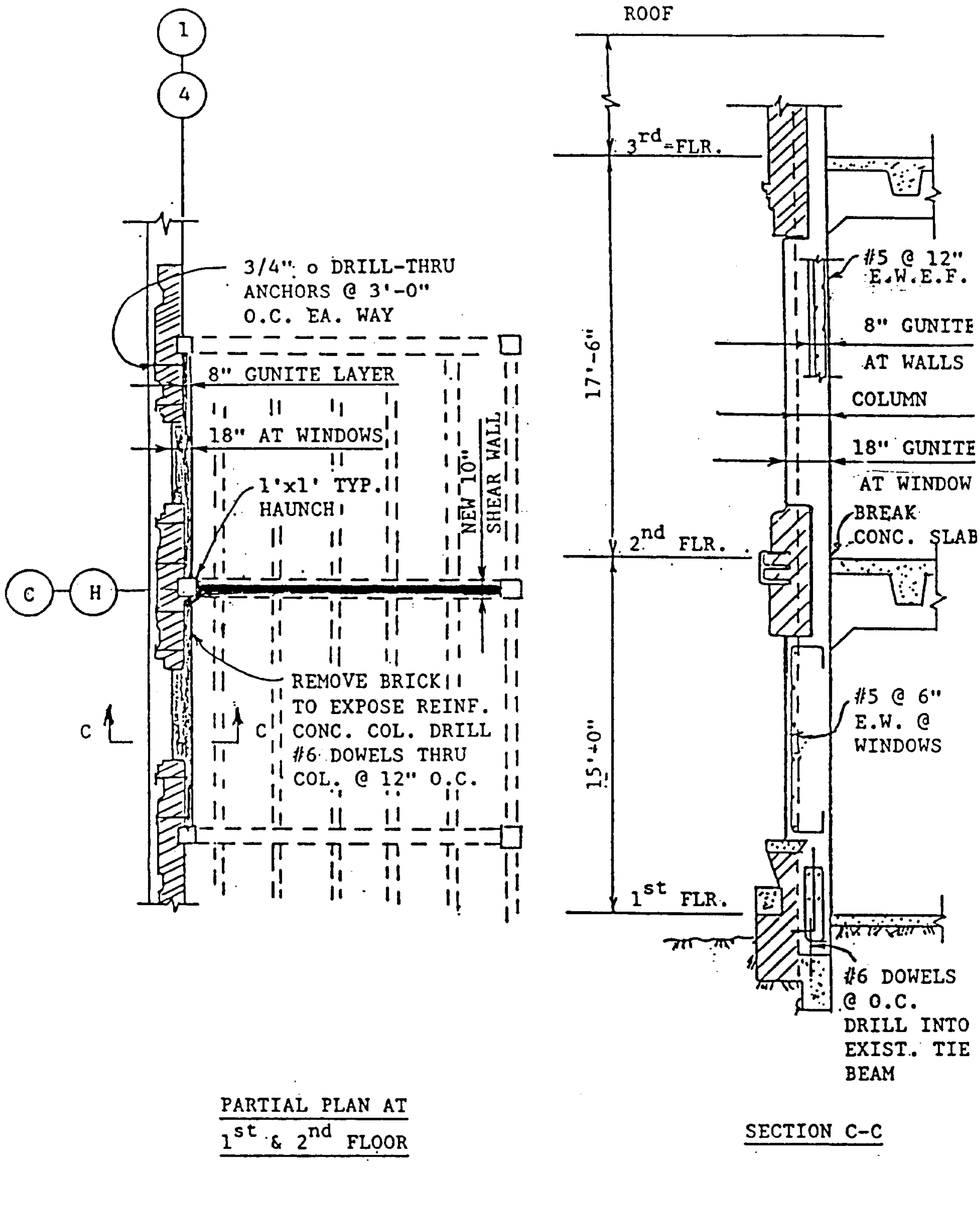
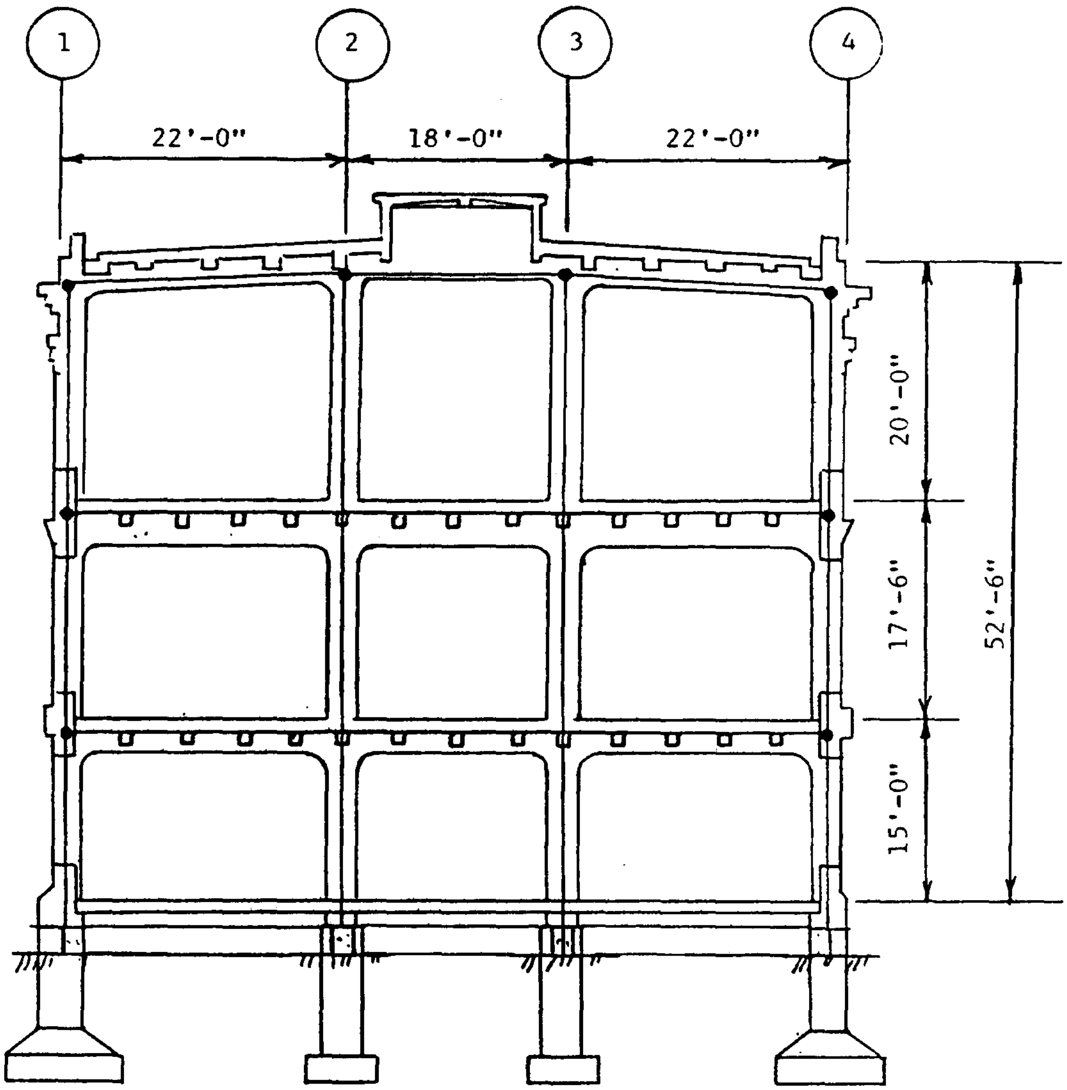


Figure F-2. Brick building with concrete framing system (Sheet 15 of 25)



LINES A, B, D, E, F, G & J

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Figure F-2. Brick building with concrete framing system (Sheet 16 of 25)

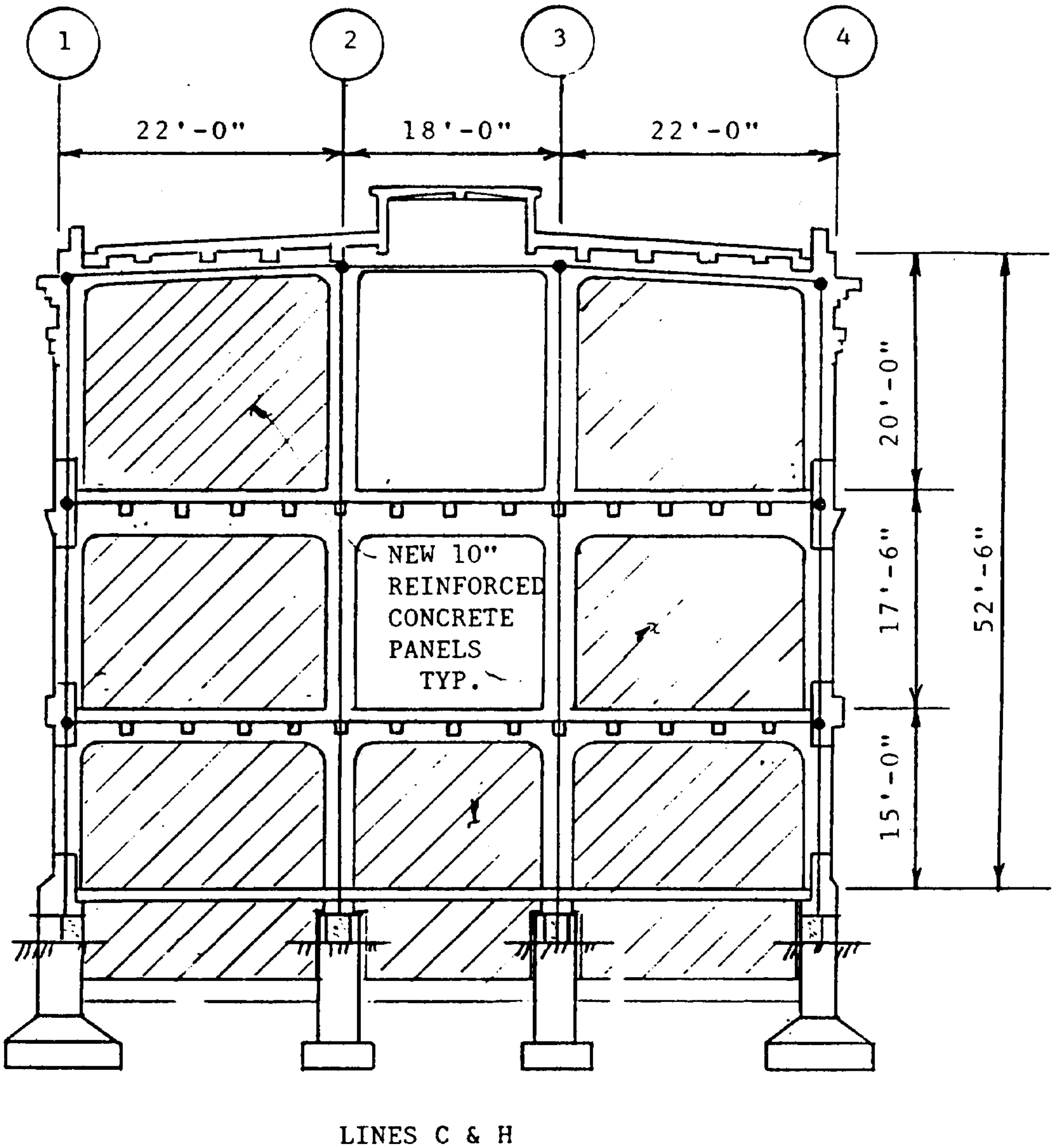

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Figure F–2. Brick building with concrete framing system (Sheet 17 of 25)

Detailed Structural Analysis to Confirm Concept. A detailed structural analysis was not necessary for the existing structure because of the negative results from the preliminary evaluation. However, a detailed analysis is now required to determine if the recommended concept will satisfy the acceptance criteria outlined on sheet 5. A modal analysis of the modified structure was made with the aid of a general computer program for static and dynamic analyses of frame and shear wall three—dimensional buildings for both the transverse and longitudinal directions. The program assumes rigid diaphragms and the roof and floor diaphragms of this modified structure essentially met this assumption. Sheets 20 and 21 indicate the SRSS of the dynamic modal responses from the computer output. Sheet 22 indicates the evaluation of the SRSS response of some representative structural elements and sheets 23 and 24 contain stress checks of selected elements for compliance with the criteria.

Torsion Forces. Due to the symmetry of the structure lateral load resisting system there is no “calculated torsion.” The “accidental torsion” is the story force times the nominal eccentricity of 5 percent of the maximum building dimension. The forces due to torsion were calculated by applying a torsional moment in each story equal to the seismic (SRSS) story shear times the “accidental” eccentricity (0.05 x 186 feet). The resulting member responses from this analysis were added to the translational member responses (SRSS) of the dynamic analyses.

Foundation Ties. The BDM (pam. 4-8a) requires that pile, caisson, and deep pier footings in seismic zones 2, 3, and 4 be interconnected by ties. In this building, the existing foundation ties are near the top of the large piers (see sheet 4 of 24) and provide questionable restraint to the timber piles. The seismic upgrading modification provides a good tie, in the plane of the new walls, for the piles on lines C and H. The significant cost and disruption of the existing building required to install new tie beams throughout the building may not be justified if it can be demonstrated that the seismic forces from EQ-II can be transmitted to the ground with the existing tie system or by passive soil pressure on the existing piers.

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Figure F-2. Brick building with concrete framing system (Sheet 18 of 25)

Results of the Confirmation Analysis. The modified structure meets all the acceptance criteria requirements for EQ-II forces except for possibly the capacity of the timber piles to support the additional loads from the new concrete walls. The capacities of the timber piles to meet the requirements for the new dead load plus live load loading criteria will need to be re-evaluated. As a result of the detailed analysis it was determined that the unreinforced brick masonry walls that were not being reinforced with gunite were deficient for seismic forces normal to the walls. These walls will either be anchored to the new concrete walls or will be provided with new vertical concrete or steel mullions between existing concrete columns for additional lateral support to meet the EQ-II acceptance criteria. Shear and flexural stresses for seismic forces parallel to the walls were found to be within the Acceptance Criteria after strengthening. An alternative modification concept was studied that provided for the anchoring of all exterior unreinforced brick masonry end walls to new reinforced concrete gunite walls placed against their interior faces in lieu of constructing the new concrete walls on Lines C and H and the additional vertical concrete mullions. This concept was rejected because it resulted in unacceptable shears in the floor and roof diaphragms and excessive overturning forces for the end walls in the transverse direction. The recommended concept could have been implemented for the entire length of the longitudinal walls thus eliminating the vertical mullions, but it is more cost effective to provide new gunite walls as required for shear resistance and new concrete mullions in the remaining portions of the existing longitudinal walls.

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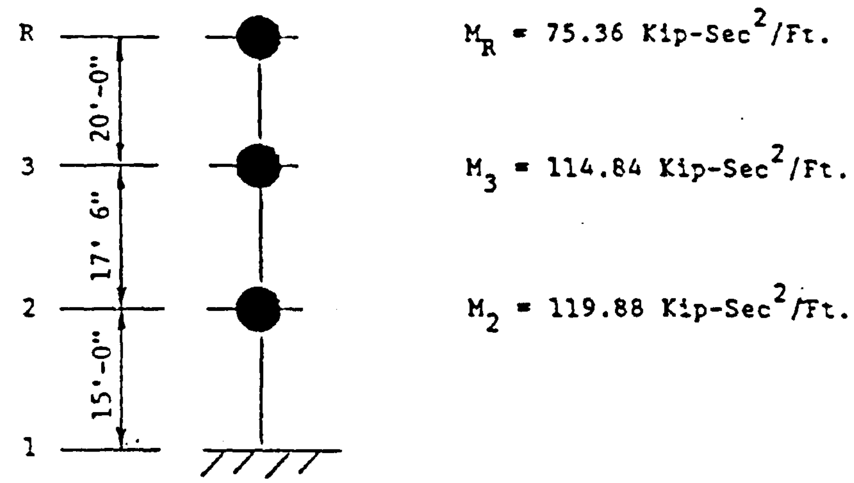
Figure F-2. Brick building with concrete framing system (Sheet 19 of 25)

MATHEMATICAL MODEL - TRANSVERSE DIRECTION



T_1 = 0.208 Sec., T_2 = 0.076 Sec., T_3 = 0.045 Sec.

EQ II - STRUCTURAL RESPONSE, TRANSVERSE DIRECTION - SRSS

LEVEL No.	STORY LOAD** F_x - Kips	STORY SHEAR V - Kips	DISPLACEMENT δ - Feet	STORY DRIFT** Δ_x - Feet
3	875	875	0.012	0.006 0.120*
2	809	1449	0.005	0.004 0.105*
1	565	1733***	0.002	0.002 0.090*

* MAXIMUM ALLOWABLE EQ II STORY DRIFT = 0.006H

** $F_x = \left[\Sigma(F_{xm})^2\right]^{\frac{1}{2}}$

$\Delta_x = \left[\Sigma(\Delta_{xm})^2\right]^{\frac{1}{2}}$

*** $C_B = V \div \Sigma W = 0.174$

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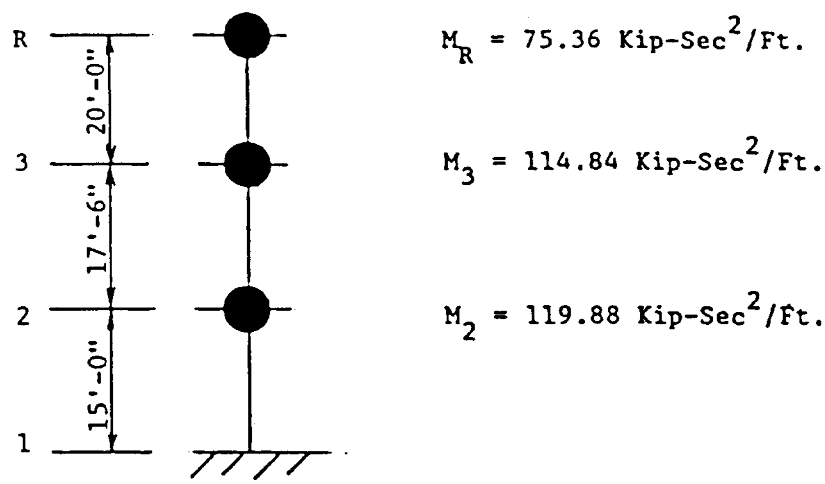
Figure F-2. Brick building with concrete framing system (Sheet 20 of 25)

MATHEMATICAL MODEL - LONGITUDINAL DIRECTION



T_1 = 0.222 Sec., T_2 = 0.068 Sec., T_3 = 0.040 Sec.

EQ II - STRUCTURAL RESPONSE, LONG. DIRECTION - SRSS

LEVEL No.	STORY LOAD** F_x - Kips	STORY SHEAR V - Kips	DISPLACEMENT δ - Feet	STORY DRIFT** Δ_x - Feet
3	873	873	0.014	0.006 0.120*
2	804	1524	0.008	0.004 0.105*
1	577	1859***	0.003	0.003 0.090*

* MAXIMUM ALLOWABLE EQ II STORY DRIFT = 0.006H

** $F_x = \left[\Sigma(F_{xm})^2\right]^{1/2}$

$\Delta_x = \left[\Sigma(\Delta_{xm})^2\right]^{1/2}$

*** $C_B = V \div \Sigma W = 0.186$

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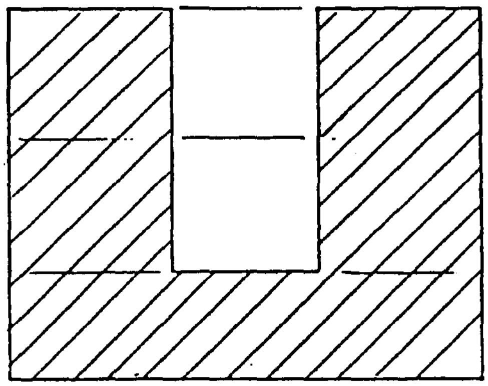
Figure F–2. Brick building with concrete framing system (Sheet 21 of 25)

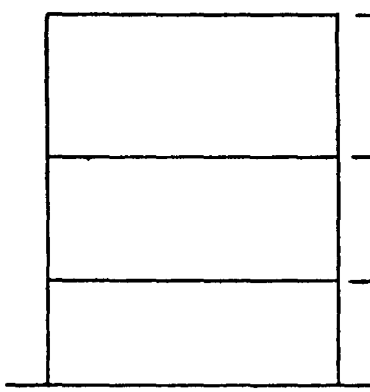
EQ II SRSS ELEMENT FORCES - TRANSVERSE DIRECTION



V (Kips)	M (Ft-Kips)
422	7880
690	19,680
843	31,572

WALLS C & H

EQ II SRSS ELEMENT FORCES - LONGITUDINAL DIRECTION



V (Kips)	M (Ft-Kips)
218	4365
381	10,812
465	17,422

WALLS ON LINES 1 & 4
(2 EACH LINE)

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Figure F-2. Brick building with concrete framing system (Sheet 22 of 25)

"ACCIDENTAL" TORSION FORCES

The "accidental" torsion is the story shear, V_x, times the nominal eccentricity of 5% of the maximum building dimension:

$$M_t = V_x \times 0.05 \times 186' = 9.3\ V_x$$

The story relative rigidity (K) of each wall line is obtained from the computer analysis.

$$\text{Torsion Shear} = \frac{Kd}{\Sigma Kd^2} \times 9.3\ V_x$$

Distribution of Forces (Frames neglected)

Roof Level

WALL LINE	REL. K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR
C	0.94	52	48.9	2542	$0.50V_T$	$0.067V_T$
H	0.94	52	48.9	2542	$0.50V_T$	$0.067V_T$
1	1.00	29	29.0	841	$0.50V_L$	$0.040V_L$
4	1.00	29	29.0	841	$0.50V_L$	$0.040V_L$
			Σ =	6766		

3rd Floor Level

WALL LINE	REL K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR
C	1.06	52	55.1	2866	$0.50V_T$	$0.069V_T$
H	1.06	52	55.1	2866	$0.50V_T$	$0.069V_T$
1	1.00	29	29.0	841	$0.50V_L$	$0.036V_L$
4	1.00	29	29.0	841	$0.50V_L$	$0.036V_L$
			Σ =	7414		

2nd Floor Level

WALL LINE	REL K	d	Kd	Kd^2	DIRECT SHEAR	TORSIONAL SHEAR
C	1.44	52	74.9	3894	$0.50V_T$	$0.074V_T$
H	1.44	52	74.9	3894	$0.50V_T$	$0.074V_T$
1	1.00	29	29.0	841	$0.50V_L$	$0.028V_L$
4	1.00	29	29.0	841	$0.50V_L$	$0.028V_L$
			Σ =	9470		

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Figure F-2. Brick building with concrete framing system (Sheet 23 of 25)

ELEMENT STRESS CHECK

Wall on Lines C & H at 2nd floor level

Moment

M_D:

EQ II Forces		19,680 ft-kips
Accidental Torsion:	19,680 x 0.069/0.50 =	2,716 " "
		22,396 ft-kips

M_u: 12,503 ft-kips

IDR: 22,396 ÷ 12,503 = 1.79 < 2.00

Shear

V_D:

EQ II Forces:		690 kips
Accidental Torsion:	690 x 0.069/0.50 =	95 "
		785 kips

V_u: 1243 kips

IDR: 785 ÷ 1243 = 0.63 < 1.25

Wall on Lines 1 & 4 at first floor level

Moment

M_D:

EQ II Forces		17,422 ft-kips
Accidental Torsion:	17,422 x 0.074/0.50 =	2,579 " "
		20,001 ft-kips

M_u: 10,088 ft-kips

IDR: 20,001 ÷ 10,088 = 1.98 < 2.00

Shear

V_D:

EQ II Forces		465 kips
Accidental Torsion:	465 x 0.074/0.50 =	69 kips
		534 kips

V_u: 802 kips

IDR: 584 ÷ 802 = 0.67 < 1.25

Roof Diaphragm

Moment

M_D = (875 kips ÷ 186 ft) x 41^2 ÷ 2 = 3950 ft-kips

M_u = 4348 ft-kips

IDR = 3950 ÷ 4348 = 0.91 < 1.50

Shear

V_D = (875 kips ÷ 186 ft) x 104 ÷ 2 = 245 kips

V_u = 292 kips

IDR = 245 ÷ 292 = 0.84 < 1.10

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Figure F-2. Brick building with concrete framing system (Sheet 24 of 25)

ELEMENT STRESS CHECK

Unreinforced Brick Masonry Wall (12" Min. Thick, $S_a = 0.36$; Max Span = 7')

Moment

M_D: (120 psf x 0.36) x $7^2 \div 8$ = 263 ft-lbs/ft

M_u: (1.6 x 7.5 psi) x 288 $in^3/12$ = 288 ft-lbs/ft

IDR: $263 \div 288 = 0.91 < 1.00$

Shear

V_D: (120 psf x 0.36) x 1.15 x $7' \div 2$ = 173 lbs/ft

V_u: (1.6 x 7.5 psi) x 144 in^2 = 1,728 lbs/ft

IDR: 173 1728 = $0.10 < 1.00$

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Figure F-2. Brick building with concrete framing system (Sheet 25 of 25)